C6.6 Piers

See the Office of Bridges and Structures web site for archived Methods Memos listed under articles in this section.

The Methods Memos for which policies have been partially revised and/or for which document references have been updated are noted as partially revised. Any obsolete Methods Memos that apply to this section are listed at the end.

C6.6.1 General

C6.6.1.1. Policy overview

~~**Methods Memo No. 181: Office Policy for Checking Piers by LRFD**~~
~~**1 December 2007**~~

C6.6.1.1.1. Frame pier

Memo 6.6.1.1.1 and 6.6.1.1.2-2011 ~ Selection of Frame or T-Pier

These changes refer back to BDM 6.6.2.6 for the vehicular collision force policy. AASHTO is likely to be changing the policy in 2011, and the office can better manage its response if the policy is in one location only.

C6.6.1.1.2. T-pier

Memo 6.6.1.1.1 and 6.6.1.1.2-2011 ~ Selection of Frame or T-Pier

These changes refer back to BDM 6.6.2.6 for the vehicular collision force policy. AASHTO is likely to be changing the policy in 2011, and the office can better manage its response if the policy is in one location only.

C6.6.1.1.3. Pile bent

C6.6.1.1.4. Diaphragm pier

C6.6.1.2. Design information

C6.6.1.3. Definitions

C6.6.1.4. Abbreviations and notation

C6.6.1.5. References

C6.6.2 Load application

C6.6.2.1. Dead

Methods Memo No. 24: Beam Design and Bearing Design, Distribution of Dead Load 2
4 September 2001

C6.6.2.2. Live

Methods Memo No. 40: Exterior Beam Distribution Factor -- LRFD
28 August 2001

C6.6.2.3. Dynamic load allowance

C6.6.2.4. Centrifugal force

C6.6.2.5. Braking force

C6.6.2.6. Vehicular collision force

C6.6.2.7. Water

C6.6.2.8. Wind

C6.6.2.8.1. Horizontal pressure on superstructure

C6.6.2.8.2. Horizontal pressure on substructure

C6.6.2.8.3. Vehicles on superstructure

C6.6.2.8.4. Vertical pressure on superstructure

C6.6.2.9. Ice

Methods Memo No. 82: Internal River Pier Ice Loads
12 January 2004 (References to the AASHTO standard specifications are obsolete.)

C6.6.2.10. Earthquake

Summary, 4 June 2008: The AASHTO LRFD seismic requirements were made considerably more complex in the 2008 interim. The 2007 specifications varied the restrained horizontal connection design forces in Seismic Zone 1 as either 0.1 or 0.2 times the tributary load, but the 2008 interim sets the horizontal connection force as either 0.15 or 0.25 times the tributary load. The 2008 interim also removes an elastomeric bearing requirement that clouded the use of a friction coefficient of 0.2.

2007 AASHTO LRFD Specifications: From the beginning of the seismic article through the Seismic Zone 1 requirements there were 10 non-blank pages organized in what seemed to be a logical sequence.

- The design process depended on the Acceleration Coefficient, A. For Iowa, A varied between 0.02 for northeastern Iowa to 0.053 in southwestern Iowa [AASHTO-LRFD Figure 3.10.2-2].
- Because all Iowa Acceleration Coefficients did not exceed 0.09, all of Iowa was in Seismic Zone 1 [AASHTO-LRFD Table 3.10.4-1].
- Site Coefficients, S, varied from 1.0 for Soil Profile Type I (best profile) to 2.0 for IV (worst profile).
- For Seismic Zone 1 with $A \leq 0.025$ and Soil Profile Type I or II the horizontal design connection force in the restrained direction was 0.1 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake [AASHTO-LRFD 3.10.9.2].
- For all other cases in Seismic Zone 1 the horizontal design connection force in the restrained direction was 0.2 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake [AASHTO-LRFD 3.10.9.2].
- For elastomeric bearings a design coefficient of friction could be assumed as 0.2 between elastomer and clean steel or concrete [AASHTO-LRFD C14.8.3.1]. That assumption was clouded by another rule regarding friction at the strength limit state [AASHTO-LRFD 14.7.6.4] (which was removed in the 2008 Interim).

2008 AASHTO LRFD Specifications: From the beginning of the seismic article through the Seismic Zone 1 requirements there are 42 non-blank pages, many of which are maps.

- The seismic design process depends on three coefficients determined from maps and three factors determined from tables. Two coefficients and two factors are applicable to Seismic Zone 1 requirements.
- Seismic Zone 1 is defined by an acceleration coefficient, $S_{D1} < 0.15$, which is determined by $S_{D1} = F_vS_1$ [AASHTO-LRFD 2008 Table 3.10.6-1, 3.10.4.2]. From the map for Iowa, S_1 varies from 0.019 in northwestern Iowa to 0.044 in extreme southeastern Iowa [AASHTO-LRFD 2008 Figure 3.10.2.1-3]. F_v varies from 0.8 for Site Class A to 3.5 for Site Class E [AASHTO-LRFD 2008 Table 3.10.3.2-3]. Site Class is defined for different soil types and layers from A to F [AASHTO-LRFD 2008 Table 3.10.3.1-1]. Site Class F is defined for peat, very high plasticity clays, or more than 120 feet of soft/medium stiff clays. For Site Class F a site specific analysis is recommended.
- All of Iowa, with the possible exception of a few sites in extreme southeastern Iowa (Lee County) generally would be classified as Seismic Zone 1 as follows: northwestern Iowa $S_{D1} = (0.019)(3.5) = 0.0665$ maximum (except Site Class F); extreme southeastern Iowa $S_{D1} = (0.044)(3.5) = 0.0154$ maximum (except Site Class F). Extreme southeastern Iowa sites with Site Class E or F soil profiles could be considered Seismic Zone 2 under a strict interpretation of the AASHTO LRFD Specifications.
- The forces to be applied to bearing connections in Seismic Zone 1 depend on the short period acceleration coefficient, $A_s < 0.05$, which is determined by $A_s = F_{pga}PGA$ [AASHTO-LRFD 2008 3.10.9.2, 3.10.4.2]. From the map for Iowa, PGA varies from 0.015 in north central Iowa to 0.040 in southeastern Iowa. F_{pga} is to be taken for the applicable Site Class [AASHTO-LRFD 2008 Table 3.10.3.2-1].
- The A_s in Iowa will vary above and below 0.05 depending on Site Class. Therefore, the horizontal design connection force in the restrained direction will be either 0.15 or 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake [AASHTO-LRFD 2008 3.10.9.2]. In general the 0.15 factor will apply for northern Iowa and the 0.25 factor will apply in southern Iowa, but the designer will need to check for the specific bridge site.
- For elastomeric bearings a design coefficient of friction may be assumed as 0.2 between elastomer and clean steel or concrete [AASHTO-LRFD 2008 C14.8.3.1]. Generally if not restrained, steel reinforced elastomeric bearings in Seismic Zone 1 need not be anchored other than by friction [AASHTO-LRFD 2008 C3.10.9.2]. However, before making the final decision regarding anchorage, the designer shall consider the rules in the steel reinforced elastomeric pads article in the manual [BDM 5.7.4.2].

C6.6.2.11. Earth pressure

C6.6.2.12. Uniform temperature

C6.6.2.12.1. General

C6.6.2.12.2. Design temperature changes

C6.6.2.12.3. Out-of-plane forces

Policy discussion with Assistant Bridge Engineer
17 January 2003

There have been two guidelines in the office for determining the locations of axial fixity for piles. The older guideline is from the 1979 version of “Design Criteria for Piers”:

> The pile length used to determine the footing rotation is assumed to be 50% and 75% of the pile length for timber and steel piling respectively.

The more recent guideline is the following:

> The percent of the pile length used to determine the footing rotation is assumed to be:
> (1) 50% when most of the pile capacity is due to friction bearing; generally wood, prestressed concrete, or steel piles not driven to bedrock, or
> (2) 75% when most of the pile capacity is due to end bearing, generally steel piles driven to bedrock.

For current design, use the more recent guideline. It is based on two concepts. The first concept is that a friction pile will transfer its load to the soil over the entire length of the pile, and thus axial deformation effectively will occur over half the pile length. The second concept is that an end bearing pile typically will transfer some load by friction and thus will not deform axially over its full length but over more length than a friction pile. The 75% value is a judgment factor.

C6.6.2.12.4. Unbalanced forces

C6.6.2.12.4.1. Bridges with integral abutments

C6.6.2.12.4.2. Bridges with stub abutments

C6.6.2.12.5. Skewed pier forces

C6.6.2.12.6. In-plane forces

C6.6.2.13. Temperature gradient

C6.6.2.14. Shrinkage

Summary 9 May 2008
Generally it is recognized that creep relieves a part of the stresses caused by shrinkage in restrained members but that it may be necessary to reinforce for shrinkage stresses. The AASHTO LRFD Specifications do not cover shrinkage in frames directly. However, for prestressed structures the specifications indicate that the designer need only consider initial elastic deformation for columns in monolithic frames and that other members will be subjected to reduced forces due to creep. Although there are no simple recommendations for shrinkage of frame piers, it is reasonable to use the AASHTO 28-day shrinkage coefficient of 0.0002 for design forces and provide the usual shrinkage and temperature reinforcement. Even though the shrinkage forces computed for 0.0002 will be reduced by the AASHTO LRFD load factor in strength limit state combinations, shrinkage forces will be included in every limit state rather than in only three load combinations under the AASHTO Standard Specifications.

Shrinkage research and standards
New York researchers (Antoni and Beal, 1971) instrumented a bridge pier to verify the AASHO 1964 specification requirements for coefficient of thermal expansion, temperature variation, and shrinkage. There were difficulties with the instrumentation, and the researchers could not separate temperature effects from shrinkage effects, but the researchers concluded that the 0.0002 shrinkage coefficient was reasonable.

In recent years there has been much study of concrete shrinkage in the hope that cracking could be reduced or eliminated. In overlays and composite structures the existing structure generally has undergone most or all of its shrinkage, and the new concrete is intended to be bonded to that structure and yet undergo shrinkage without cracking. Because of the infinite variety of concrete mixtures, the wide variety of structural conditions, the related factors such as thermal expansion/contraction and creep, and the differences between tension and compression creep, researchers have not found any simple and definitive answers to questions regarding shrinkage.

One researcher, however, has suggested that at two years creep reduces sustained tensile stresses by about one-third (Alexander 2005, 2007). The researcher also suggested that the designer provide reinforcement for tension caused by shrinkage.

The AASHO/AASHTO Standard Specifications have included the concrete shrinkage coefficient of 0.0002 since 1941. Specifications of that vintage had no commentary, so the source of the coefficient is unknown, except that New York researchers attribute the coefficient to experimental data from small laboratory specimens (Antoni and Beal, 1971).

The ISU strip seal report (Bolluyt et al. 2001) concluded that 0.0002 was an appropriate shrinkage value for concrete decks on prestressed beams or steel girders, although some of the bridges studied indicated more or less shrinkage.

The report did not recommend any modification factors. (There was no proposal to use modification factors with thermal expansion/contraction either, but there was a recommendation to increase the temperature range for computing concrete bridge thermal movements.)

Of the surrounding six states, only Nebraska (0.0002) and Wisconsin (0.0003) include concrete shrinkage in their strip seal designs. In the ISU report there is no indication of whether the states include shrinkage in frame pier design.

The AASHTO LRFD equations for shrinkage and compressive creep [AASHTO-LRFD 5.4.2.3] show that both develop in the same pattern and, assuming the strain in concrete is about 10% of the crushing strain (among other assumptions), the creep and shrinkage curves approximately match. Using the shrinkage equation with assumptions, shrinkage can be computed of about 0.0007 at one year and 0.0008 at 2.5 years. The AASHTO LRFD Specifications, however, generally indicate shrinkage of 0.0002 at 28 days and 0.0005 at one year [AASHTO-LRFD 5.4.2.3.1].

The AASHTO LRFD Specifications do require that effects of prestressing deformations on adjoining elements of the structure be evaluated [AASHTO-LRFD 5.9.2]. These deformations could be considered to be similar to shrinkage deformations. Additionally the specifications state the following:

> In monolithic frames, force effects in columns and piers resulting from prestressing the superstructure may be based on the initial elastic shortening.
>
> For conventional monolithic frames, any increase in column moments due to long-term creep shortening of the prestressed superstructure is considered to be offset by the concurrent relaxation of deformation moments in the columns due to creep in the column concrete.
>
> The reduction of restraining forces in other members of a structure that are caused by the prestress in a member may be taken as….

Considering the statements above, it is prudent to include some value of shrinkage in design. The statement regarding use of initial elastic shortening in monolithic frames could be taken to suggest the shrinkage coefficient of 0.0002 at 28 days. The next AASHTO statement could be taken to suggest that relaxation due to creep will relieve additional column moments due to shrinkage after 28 days. The AASHTO slowly imposed deformation equation (not copied above) with assumptions indicates that a 100-kip force, such as a cap force based on shrinkage, for time from 28 days to 896 days would reduce to 43 kips, less than half the initial force.

References

Alexander, S.J. (2007). “The Importance of Time in Understanding Concrete Behavior,” *Conference on Structural Implications of Shrinkage and Creep of Concrete, SP-246*, pp 283-289.

Alexander, S.J. (2005). “Managing Deflection, Shortening and Cracking Arising from Restrained Contraction,” *Shrinkage and Creep of Concrete, SP 227*, pp 1-20.

American Association of State Highway Officials (AASHO). (1941) *Standard Specifications for Highway Bridges, Third Edition,* American Association of State Highway Officials, Washington, DC.

Antoni, C.M. and D.B. Beal. (1971). *Temperature and Shrinkage Stresses in a Concrete Bridge Pier, Research Report 69-6*, Engineering Research and Development Bureau, New York Department of Transportation, Albany, New York.

Bolluyt, J.E., V.B. Kau, and L.F. Greimann. (2001). *Performance of Strip Seals in Iowa Bridges, Pilot Study, Final Report Project TR-437,* College of Engineering, Iowa State University, Ames, Iowa.

C6.6.2.15. Creep

C6.6.2.16. Locked-in force

C6.6.2.17. Settlement

C6.6.2.18. Friction

C6.6.2.18.1. Out-of-plane forces

C6.6.2.18.2. Unbalanced forces

C6.6.2.18.2.1. Bridges with integral abutments

C6.6.2.18.2.2. Bridges with stub abutments

C6.6.2.18.3. Skewed pier forces

C6.6.2.19. Vessel collision

C6.6.3 Load application to structure

C6.6.3.1. Load modifier

C6.6.3.2. Limit states

Memo 6.6.3.2-2010: Service I Limit State

With use of RCPIER software the Service I load combination may appear to control pile design, however there is no need to consider Service I for strength design of piles. Service I may be used to check settlement of piles.

C6.6.3.3. Longitudinal and transverse forces transmitted through bearings

C6.6.3.3.1. Elastomeric bearings

C6.6.3.3.2. Fixed bearings and keyed-in concrete diaphragms

C6.6.3.3.3. Friction-acting bearings

C6.6.3.4. Longitudinal and transverse forces for non-skewed piers

C6.6.3.5. Parallel and perpendicular forces for skewed piers

C6.6.4 Pier Components and details

~~**Methods Memo No. 181: Office Policy for Checking Piers by LRFD**~~
~~**1 December 2007**~~

C6.6.4.1. Frame piers and T-piers

Memo 6.6.4.1-2010 ~ Location of fixity for frame and T-piers on pile footings

Column analysis and design is sensitive to slenderness, and the designer should not model a column taller than the structural configuration allows. Although it would be acceptable to model a pier column as fixed at 2 feet (600 mm) below top of footing or at mid-depth of footing, the column is restrained significantly at the top of footing, and the designer should assume fixity at that elevation to minimize slenderness. Available pier software makes it relatively easy to analyze alternate models, and for pile and footing design the designer should assume pier columns extend to bottoms of footings.

Methods Memo No. 16: Use of Higher Strength Concrete
21 March 2001

Methods Memo No. 64: Removal of Corrosion Inhibitor Option on Columns
18 June 2002

C6.6.4.1.1. Pier cap

C6.6.4.1.1.1. Analysis and design

Memo 6.6.4.1.1.1-2011: Longitudinal Reinforcement for Shear

Equation 5.8.3.5-1 will compute additional longitudinal reinforcement anywhere along a member where there is non-zero factored shear. For a typical frame pier cap, RCPIER will provide the area of the additional reinforcement at sections along the cap selected by the engineer.

The AASHTO LRFD Specifications, however, permit the designer to neglect the additional reinforcement at certain locations as follows:

> "The area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone. This provision applies where the reaction force or the load introduces direct compression into the flexural compression face of the member."

For direct loading this exception is shown graphically in AASHTO LRFD Figure C5.8.3.5-2 extending a distance $d_v \cot\theta$ either side of the maximum moment. For the typical frame pier cap with two cantilevers, the exception will apply at each support column and at each beam bearing where M_u is maximum (V_u changes sign) in the cap. The designer should make use of this exception.

Memo 6.6.4.1.1.1-2010: Longitudinal Reinforcement for Shear

The requirement in the AASHTO LRFD Specifications for longitudinal reinforcement for shear [AASHTO LRFD 5.8.3.5] was added to the specifications at the same time as modified compression field theory. The requirement substitutes for ACI Code and AASHTO Standard Specifications rules for end supports, points of inflection, and cutoffs in tension zones. Because AASHTO LRFD Figure C5.8.3.5-2 does not show a cantilever condition, designers have asked whether the requirement would increase the top steel in pier cap cantilevers. Numerical examples indicated that the increase could be significant.

Examples in the literature are inconclusive, but a cantilever truss analogy worked in the office similar to the simple span truss analogy in *Reinforced Concrete Mechanics and Design, Fifth Edition* by Wight and MacGregor showed that the top steel in a pier cap cantilever need not be increased beyond that required for moment at the pier column. As in AASHTO LRFD Figure 5.8.3.5-2 the demand for longitudinal reinforcement did increase away from the support over the entire distance to the free end. For a typical pier cap cantilever design without bar cutoffs the top reinforcement is not affected.

Partially revised: Methods Memo No. 8: Pier Cap Design Shear Stirrup Spacing (Frame pier cap height has been modified in the manual.)
9 April 2001

Methods Memo No. 16: Use of Higher Strength Concrete
21 March 2001

Methods Memo No. 7: Using hooks near cap ends for development of flexural reinforcing steel
19 January 2001

Partially revised: Methods Memo No. 211: Office Guidelines for Mass Concrete and Temperature and Shrinkage Reinforcing
1 September 2009

C6.6.4.1.1.2. Detailing

Methods Memo No. 107: Integral Abutment and Pier Cap Detailing
6 June 2005

Methods Memo No. 196: Pier Pedestal Guidelines for Aesthetic Piers (LRFD Bridge Design manual 6.6.4.1.1.2)
1 January 2008

C6.6.4.1.2. Pier column

C6.6.4.1.2.1. Analysis and design

Memo 6.6.4.1.2.1-2011 ~ Minimum T-Pier Column Reinforcement

In order to limit longitudinal reinforcement in large pier columns, designers in the past have been permitted to consider a reduced effective cross section area, if that reduced area had a minimum of 1% reinforcement. To be in compliance with the AASHTO LRFD Specifications in 2001, with Methods Memo No. 18 the office additionally restricted the minimum reinforcement to 0.7% of the actual gross area.

Since 2001 Article 5.7.4.2 in the AASHTO specifications has been revised to consider f'_c and f_y in setting the minimum reinforcement for the gross area. Equation 5.7.4.2-3 now limits the minimum reinforcement to 0.79% of the gross column area for $f'_c = 3.5$ ksi and $f_y = 60$ ksi. For $f'_c = 4$ ksi and $f_y = 60$ ksi the minimum reinforcement must be 0.90% of the gross area. In some cases, even with one of these reductions, there is difficulty in fitting the longitudinal reinforcement at the perimeter of a large pier column.

For bridges in Seismic Zone 1 the current AASHTO LRFD Specifications also permit a reduced effective column area provided that the reinforcement is the larger of 1% for the reduced area or the amount from Equation 5.7.4.2-3 for the reduced area. However there is no minimum limit for the reduced effective area. Based on previous successful experience the office will allow the designer to design for a reduced effective area that as a minimum is 50% of the actual area. This minimum area follows ACI 318-08 Article 10.8.4.

Memo 6.6.4.1.2.1/6.6.4.1.3.1-2010: Structural Models for Piers

Initially with the use of RCPIER the office recommended that designers model piers with columns fixed 2 feet (600 mm) below the top of footing. This modeling applied for both pier frame and foundation and made it possible to run the software only once. However, this modeling resulted in columns 2 feet (600 mm) taller than they actually were and, in some cases, led to a significant increase in moment magnification.

In order to minimize moment magnification the office now is giving designers the option of fixing the columns at the tops of the footings. This option will require the designer to run the software twice, once for the pier columns and cap and once for the footings and piles.

Memo 6.6.4.1.2.1-2010-II: Location of Fixity for Frame and T-Piers on Pile Footings

No commentary.

Memo 6.6.4.1.2.1-2010: Pier Column Shear Reinforcement

Pier columns subjected to significant lateral loads need to be designed for shear as well as axial loads. The text of the manual was revised to alert the designer to the need to meet both shear stirrup and column tie requirements.

Methods Memo No. 107: Integral Abutment and Pier Cap Detailing
6 June 2005

Partially revised: Methods Memo No. 5: Maximum T-Pier Heights Based on Kl/r
24 January 2001 (Much of this memo is obsolete, however the options still are valid.)

Partially revised: Methods Memo No. 211: Office Guidelines for Mass Concrete and Temperature and Shrinkage Reinforcing
1 September 2009

C6.6.4.1.2.2. Detailing

Methods Memo No. 64: Removal of Corrosion Inhibitor Option on Columns
18 June 2002 (Supersedes Methods Memo No. 42, which has been moved to the appendix for this commentary section)

Methods Memo No. 75: End Bar Clearances for Horizontal Construction Joints
6 July 2005

Methods Memo No. 204: General Note on Keyway Dimensions
1 October 2008

Partially revised: Methods Memo No. 69: Epoxy Coated Spirals
10 August 2004 (Portion in curly brackets {} regarding lap length superseded by manual text)

Methods Memo No. 109: Reinforcement Placement in Round Columns
3 March 2006

C6.6.4.1.3. Pier footing

C6.6.4.1.3.1. Analysis and design

Memo 6.6.4.1.2.1/6.6.4.1.3.1-2010 Structural Models for Piers
The option of using two models in RCPIER reduces the effect of moment magnification for design of pier columns. See C6.6.4.1.2.1 for additional information.

Partially revised: Methods Memo No. 211: Office Guidelines for Mass Concrete and Temperature and Shrinkage Reinforcing
1 September 2009 (Item 4. was edited for clarity 24 November 2009.)

Methods Memo No. 192: LRFD Office Guidelines for Temperature and Shrinkage Reinforcing in Pier Footings
1 March 2008

Partially revised: Methods Memo No. 3: Punching Shear and Wide Beam Shear
21 March 2001 (Revised 29 January 2003. For LRFD neglect references to AASHTO Standard Specifications.)

Methods Memo No. 145 gave rules for checking piles at design and check scour conditions under the AASHTO Standard Specifications. The rules were based on service load design (SLD) and have been updated for the AASHTO LRFD Specifications. The following is a brief summary of the LRFD specifications regarding scour.

Service limit state

- 10.5.2: Consider foundation movements, including movement at the design scour condition.
- 10.5.5.1: Use a resistance factor of 1.0 for deflection at the design scour condition.
- 10.7.2.1: Evaluate overall stability for loss of support due to scour.

Strength limit state

- 10.5.3.1: Consider loss of support due to scour at the design flood.
- 10.5.5.2.1: Factored foundation resistance after design flood scour must be greater than factored load with scoured soil removed.
- 10.5.5.2.3: Use resistance factors specified for geotechnical resistance, structural resistance, and drivability analysis.
- 10.7.3.6: Select pile penetration to be adequate after scour. Consider debris loads during flood event.

Extreme event limit state

- C10.5.4: Design for check flood scour.
- 10.5.5.3.2: Nominal resistance ($\varphi = 1.0$) after check flood scour is to be adequate for unfactored strength limit state loads. For uplift take $\varphi = 0.8$ or less. Consider debris loads during the check flood.
- 10.7.4: Use check flood and resistance factors from 10.5.5.3.2.

For geotechnical design the procedures in Methods Memo No. 145 are superseded by the AASHTO LRFD Specifications, which are more liberal for check scour. After severe flooding, scour will be evaluated by Iowa DOT bridge inspection teams under the Scour Watch program, and therefore, under check scour conditions, safety will be assured by design, field inspection, and bridge closures.

For structural design, Structural Resistance Level - 2 (analogous to 9 ksi) at KL/r of 80 for design scour in Methods Memo No. 145 has been taken as the basic condition and extrapolated under LRFD to Structural Resistance Level – 3. For check scour the maximum slenderness has been taken at 120 (the maximum for a main compression member) for all Structural Resistance Levels. This will result in a small apparent margin of safety for piles under check scour, more than required by the AASHTO LRFD Specifications, which will allow some capacity for bending of the pile under stream flow pressure. If the superstructure is partially or fully inundated by the 100-year flood the designer will be required to check the 500-year flood condition at the extreme event limit state as discussed in BDM 6.6.2.7.

For Table 6.6.4.1.3.1-1 the maximum slenderness ratios for design scour were determined to provide an apparent margin of safety, γ/φ (load factor/resistance factor), of about 3.4 for a compression load. The maximum slenderness ratio of 120 [AASHTO-LRFD 6.9.3] at the check scour condition results in an apparent margin of safety of about 1.4, minimum, for compression load. Because the basic column stability formula was changed from the AASHTO Standard Specifications to the AASHTO LRFD Specifications, the apparent margins of safety in the two specifications cannot be compared directly.

References

Galambos, T.V., Ed. (1998). *Guide to Stability Design Criteria for Metal Structures, Fifth Edition,* John Wiley & Sons, Inc., New York, NY.

Tide, R.H.R. (2001). “A technical note: derivation of the LRFD column design equations,” *Engineering Journal,* third quarter, 137-139.

Example: Scour analysis and design, pier pile, LRFD

Given: Pier pile, Grade 50, HP 10x57
Unfactored axial compression load at the strength limit state, P = 135 kips
Factored axial compression load at the strength limit state, P_u = 210 kips at SRL-2
(This factored load is less than the maximum factored load that would be permissible at SRL-2, (0.6)(365) = 219 kips [BDM Table 6.2.6.1-1].)
Elevation, bottom of footing: 640 feet
Elevation, design scour: 633 feet
Water velocity, design flood: 5 feet/sec
Elevation, check scour: 630 feet
Soil profile: 10 feet silty sand, N = 8

20 feet firm glacial clay, N = 11
60 feet very firm sandy glacial clay, N = 25

Determine pile length considering geotechnical resistance with design scour [BDM 6.2.7]. This condition will control rather than the condition without scour. Also, determine driving resistance in scourable material.

Required geotechnical resistance = P_u/φ_c = 210/0.725 = 289.7 kips

Cutoff after driving	1 foot		
Pier embedment	1 foot		
Silty sand	10 feet (less scour)	(3)(1.2) =	3.6 kips
Firm glacial clay	20 feet	(20)(2.8) =	56.0 kips
End bearing in very firm glacial clay		(16.8)(2) =	33.6 kips
Very firm glacial clay	50 feet	(50)(4.0) =	200.0 kips
Total	82 feet, round to <u>85 feet</u>		293.2 kips > 289.7 kips, OK

Theoretical driving resistance in scoured material

Silty sand	7 feet	(7)(1.2) =	8.4 kips

Check geotechnical resistance with check scour [BDM 6.2.7]

Required geotechnical resistance = P/φ_c = 135/1.00 = 135.0 kips

Cutoff after driving	1 foot		
Pier embedment	1 foot		
Silty sand	10 feet (less scour)	(0)(1.2) =	0.0 kips
Firm glacial clay	20 feet	(20)(2.8) =	56.0 kips
End bearing in very firm glacial clay		(16.8)(2) =	33.6 kips
Very firm glacial clay	50 feet	(50)(4.0) =	200.0 kips
Total			289.6 kips > 135.0 kips, OK

Check structural resistance at design scour [BDM Table 6.6.4.1.3.1-2]

Pile length = (640-633) + 4 = 11 feet < 16.0 feet, OK

Check structural resistance at check scour [BDM Table 6.6.4.1.3.1-2]

Pile length = (640-630) + 4 = 14 feet < 24.1 feet, OK

Determine driving resistance values to be included in plan note.

Dead and live load	= (75)(210/219) =	71.9 tons
Scourable material	= (71.9)(8.4/289.7) =	2.1 tons
Total		74.0 tons

CADD Note E834 on plans (assuming that the pier pile in this example was the controlling pile):

PIER PILES ARE DESIGNED TO ACCOMMODATE THE ABSENCE OF SCOURABLE SOILS ABOVE THE 100 YEAR SCOUR ELEVATION SHOWN IN THESE PLANS. PILES SHALL BE DRIVEN TO 74.0 TONS BASED ON THEORETICAL DRIVING RESISTANCE. THIS INCLUDES 2.1 TONS OF RESISTANCE IN THE SCOURABLE LAYERS, AND 71.9 TONS RESISTANCE FOR DEAD AND LIVE LOAD BEARING CAPACITY.

Methods Memo No. 117: Pile Cutoff for Battered Piles
20 July 2005

Methods Memo No. 9: Battered Pile Capacity and Lateral Load Capacity for Pier Design
9 April 2001

Methods Memo No. 2: Top Bar Factor for Footing Reinforcing Steel Development Length
2 January 2001

C6.6.4.1.3.2. Detailing

Methods Memo No. 50: Detailing of Footing to Column Rebar
28 September 2001

C6.6.4.1.4. Seal coat

Methods Memo No. 21: Use of "Excavate and Dewater" Bid Item
5 February 2002

C6.6.4.1.4.1. Analysis and design

For this LRFD manual the loads, resistances, and factors were calibrated directly from service load design (SLD) in Methods Memo No. 154. Computations and checks under either SLD or LRFD should give the same results.

Methods Memo No. 21: Use of "Excavate and Dewater" Bid Item
5 February 2002

Methods Memo No. 154: Design of Cofferdam Seal Coat
17 November 2006

C6.6.4.1.4.2. Detailing

C6.6.4.2. Pile bents and diaphragm piers

C6.6.4.2.1. Analysis and design

Partially revised: Methods Memo No. 19: Guidelines for Fully Encased Pile Bents for Reinforced Concrete Slab Bridge (Revised in text of manual 22 December 2008)
18 April 2001

C6.6.4.2.1.1. Steel H-piles

Steel H-pile, pile bent example:

Given: Three-span (67'-6, 80'-0, 67'-6) PPCB bridge over gully without stream, 218 feet long with 40-foot roadway and 15-degree skew
Steel H-piles
Factored total DC + DW + LL + IM load at pile bent = P_u = 1800 kips
Maximum height to underside of non-monolithic cap = 18 feet
Ground slope along pile bent 1:30 maximum
Soil profile: 0-3 feet: top soil
3-75 feet: firm - very firm glacial clay with N = 14

(1) Determine size and number of H-piles for structural condition above ground

Check applicability of Bridge Design Manual simplified method [BDM 6.6.4.2.1]
Bridge length: 218 feet < 250 feet …OK
Roadway width: 40 feet < 44 feet …OK
Skew: 15 degrees < 45 degrees …OK
Thermal expansion for steel substructure [AASHTO-LRFD 3.4.1, p3-11]:
(80’/2)(12)(0.000006)(50)(1.0) = 0.144 inches < 0.45 inches …OK
Ground slope: 1:30 < 1:10 …OK

Choose pile shape [BDM Table 6.6.4.2.1.1]
Try HP 12x53 because H = 18 feet (> 16 feet for HP 10x57)

Determine number of piles [BDM Table 6.6.4.2.1.1]
$n = P_u/\varphi_c P_n$ = 1800/(0.7)(192) = 13.4, try 14 piles
Spacing = 3’- 0 3/4 > 2.50 feet …OK [spacing from H40-06 series sheet H40-48-06]

(2) Check HP 12x53 structural condition in the ground

Per pile, P_n = 224 kips [BDM Table 6.2.6.1-1]
For bent, $P_r = n\varphi_c P_n$ = (14)(0.6)(224) = 1881.6 kips > P_u = 1800 kips …OK

(3) Check geotechnical condition, determine pile length, and write plan note.

Compute required nominal geotechnical resistance per pile.
$P_n = P_u/\varphi_c n$ = 1800/(0.725*14) = 177.3 kips

Determine pile length.

Cutoff	1 foot		
Cap embedment	1.5 feet		
H	18 feet		
Encasement	3 feet		
Firm - very firm glacial clay	27 feet	(27)(3.2)= 86.4	
Subtotal			86.4 kips

Required additional resistance = 177.3-86.4 = 90.9 kips
End bearing is not to be considered [BDM 6.2.7].
Required length in firm -very firm glacial clay = 90.9/4.8 = 18.9+ feet

Firm - very firm glacial clay	19.0 feet	(19.0)(4.8)	= 91.2
Totals	69.5 feet, round to 70 feet		177.6 kips

Check driven length: 27+19 = 46 feet > 21 feet …OK

Compute theoretical driving resistance values to be included in plan note.

Dead, live, and dynamic load = (46)(13.4/14) = 44.0 tons

or (46)(1800/14*0.7*192) = 44.0 tons

<u>CADD Note E720 on plans</u> [BDM 11.8.2]: THE DESIGN BEARING FOR THE PIER PILES IS 44.0 TONS.

<u>Use 14 – HP 12x53 piles, 70 feet long.</u>

Because the piles are designed to Structural Resistance Level – 1 (SRL-1) no drivability analysis is required [BDM 6.2.6.1].

C6.6.4.2.1.2. Prestressed concrete piles

C6.6.4.2.1.3. Concrete-filled steel pipe piles

C6.6.4.2.2. Detailing

Obsolete: Methods Memo No. 1: Footing Elevation for Scour and Allowable Unsupported Pile Lengths
23 March 2001

Obsolete: Methods Memo No. 4: Individual Punching Shear and Beam Shear
19 January 2001

Obsolete: Methods Memo No. 6: Pier Cap Design, Shear Stirrup Spacing
9 April 2001

Obsolete: Methods Memo No. 18: Minimum Steel for Column Design
18 April 2001 (Superseded by AASHTO-LRFD 5.7.4.2 provisions for minimum reinforcement)

Obsolete: Methods Memo No. 42: Use of Corrosion Inhibitor Option on Columns
30 August 2001 (Superseded by Methods Memo No. 64)

Obsolete: Methods Memo No. 72: Application of Wind Load to Substructure
27 August 2002

Obsolete: Methods Memo No. 145: Pier Foundation Design and Check for Scour Conditions
4 September 2007 (Partially supersedes Methods Memo No. 1 and has been superseded by manual changes for LRFD 24 December 2008)

Obsolete: Methods Memo No. 181: Office Policy for Checking Piers by LRFD
1 December 2007

Appendix for technical documents

See separate document.